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7. Author(s) Frank P. Mancini and Charles E. O'Bannon					
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16. Abstract  A large scale soil stabilization project was initiated in July 1973 by the Arizona Department of Transportation. This project was sponsored by the Federal Highway Administration.  A 500-foot section, which is part of a 1300 foot cut on I-40 35 miles east of Holbrook, Arizona, was selected as the test site. The natural soil in the area is Chinle clay, a highly expansive Montmorillonite clay.  The stabilization technique used on the Chinle clay was treatment with a 0.4N solution of KCl and electrochemical action.  Results show that this method of soil stabilization produced a moderate decrease in the swelling characteristics of the Chinle clay, i.e., about a 50% decrease in expansive pressure and a 36% decrease in percent swell.  Moreover, the electron micrographic and x-ray diffraction data have indicated some probable causative factors for the decrease in the swelling characteristics of the Chinle clay.					
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## PREFACE

The contents of this report represent over two year's work conducted by the Arizona Department of Transportation, Research Section. This work entails the development and analysis of a field procedure for the stabilization of Chinle Clay using electro-chemical methods.

## ACKNOWLEDGEMENTS

The principal researchers on this study feel a deep sense of appreciation and gratitude to many individuals and Organization for the excellent, and continuing spirit of close cooperation that existed during the 2 year period of this study.

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Assistant Engineer of Materials

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Soils Engineer

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Civil Engineer I

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## TABLE OF CONTENTS

Abstract . . . . .	i
Preface . . . . .	ii
Table of Contents. . . . .	iii
List of Figures. . . . .	v
List of Tables . . . . .	viii

CHAPTER	PAGE
I. Introduction . . . . .	1
Sponsorship . . . . .	1
Nature of and Approach to Problem . . . . .	1
Object of Research . . . . .	4
Scope of Report . . . . .	4
II. Geology and Preliminary Sampling of the Areas . . . . .	5
III. Physicochemical Aspects . . . . .	10
Base Exchange . . . . .	12
Classical Modes of Swelling Mechanisms. . . . .	15
Clay Stabilization by Electro-Osmosis & Base Exchange of Ions .	21
Miscible Displacement . . . . .	23
Introduction . . . . .	23
Solute Displacement . . . . .	23
Experimental Procedure. . . . .	24
Breakthrough Curves . . . . .	27

CHAPTER	PAGE
IV. Laboratory work . . . . .	35
General . . . . .	35
Site Selection and Sampling . . . . .	35
Sample Preparation . . . . .	37
Soil properties of untreated material . . . . .	41
Expansive pressure and percent swell tests . . . . .	41
Determination of optimum gm KCL/gm clay . . . . .	44
V. Field work . . . . .	46
General . . . . .	46
Site selection and sampling . . . . .	46
Field Test . . . . .	46
Discussion of Operational Characteristics of E. O. Soil Stabilization . . . . .	67
VI. Results and analysis of field test . . . . .	76
Introduction . . . . .	76
X-ray diffraction analysis . . . . .	76
Conclusions from X-ray diffraction data . . . . .	86
Transmission Electron Microscope . . . . .	89
Conclusions from Transmission Electron Microscope Data .	96
Expansive pressure and % swelling of Post-Test samples .	102
Conclusions from Soil Test Data . . . . .	107
VII. Monetary & Energy Expenditures for Electrochemical Operation . . . . .	109
VIII. Nuclear Moisture Monitoring . . . . .	113
IX. Summary, Conclusions and Recommendations . . . . .	120

# LIST OF FIGURES

FIGURE		PAGE
1	Major Chinle Outcrops . . . . .	3
2	Location of Test Site . . . . .	6
3	Soil Profile . . . . .	9
4	Surface Symmetries . . . . .	18
5	b-dimension vs water content . . . . .	19
6	Effect of Cation on b-dimension . . . . .	20
7	Miscible Displacement . . . . .	26
8-a	Miscible Displacement . . . . .	28
8-b	Break-through Curve . . . . .	29
9 & 10	Miscible Displacement . . . . .	31
11	Displacement of KCL Solution in soil . . . . .	33
12	Test Site Layout . . . . .	36
13	Sampling Plan Layout . . . . .	38
14	Flow Chart . . . . .	39
15	Soil Grain size distribution . . . . .	40
16	Expansive pressure vs % KCL . . . . .	45
17	Pre-test Sampling Plan . . . . .	46
18	E.O. Electrical Circuit . . . . .	49
19	Cathode Section Shown . . . . .	50
20	E.O. Electric field . . . . .	51
21	Auger holer shown . . . . .	53
22	27" long steel Sleeve shown . . . . .	54
23	Auger hole drilling . . . . .	56



FIGURE		PAGE
24	Partial view of test site . . . . .	57
25	3/4 view of test site . . . . .	57
26	Cathode Trenching Operation . . . . .	59
27	Preparation of snode trenches . . . . .	59
28	60 KW D.C. Generator . . . . .	60
29	3/4 view of Generator . . . . .	60
30	Transverse E.O. Cables . . . . .	61
31	Cathode welding operation . . . . .	61
32	2500 gallon tanker . . . . .	62
33	KCL Solution preparation . . . . .	62
34	Solution delivery system . . . . .	64
35	Solution delivery system . . . . .	64
36	On site solution input . . . . .	65
37	Close up of solution hose and nozzle . . . . .	65
38	Post test sampling plan . . . . .	66
39	Pre-test, X-ray diffraction, test site material . . . . .	79
40	Post-test, X-ray diffraction, test site material . . . . .	80
41	X-ray diffraction, sample material not from site . . . . .	81
42	X-ray diffraction, sample materia not from site . . . . .	82
43	X-ray diffraction, sample material not from site . . . . .	83
44	X-ray diffraction, sample material not from site . . . . .	84
45	Electron diffraction pattern from treated test site . . . . .	91
46	Electron diffraction pattern from treated test site . . . . .	92

FIGURE	PAGE
47	Electron diffraction pattern from untreated test site . . . 93
48	Electron diffraction pattern from untreated test site . . . 94
49	Electron micrograph sample preparation . . . . . 97
50	Wyoming bentonite electron diffraction pattern . . . . . 98
51-a	Old E.O. site electron diffraction pattern . . . . . 99
51-b	Electron transmission micrograph . . . . . 101
51-c	Electron transmission micrograph . . . . . 101
52	ASU designed expansometer, full view . . . . . 105
53	ASU designed expansometer, partially disassemble . . . . . 105
54	ASU designed expansometer, fully disassemble . . . . . 106
55	ASU designed expansometer, full view . . . . . 106
56	Nuclear moisture monitor lay-out . . . . . 114
57	Typical access tube installation . . . . . 115
58	K <sup>+</sup> ion concentration vs distance from auger hole . . . . . 116
59	CL-ion concentration vs distance from auger hole . . . . . 117
60	Typical diffusion curves under action of constant force . 118
61	Electrochemical soil stabilization flow chart . . . . . 127

## LIST OF TABLES

TABLE #		PAGE
1	Untreated soil index properties . . . . .	42
2	Untreated expansive pressures and percent swell . . . .	43
3	X-ray diffraction data of treated and untreated soil .	85
4	Engineering properties of treated soil . . . . .	103
5	Post test samples . . . . .	108

## CHAPTER 1

### INTRODUCTION

#### Sponsorship

This report is based on work performed by the Arizona Highway Department, under contract with the Department of Transportation to investigate "Electro-Chemical Hardening of Expansive Clays". Funds for the project were provided by the Federal Highway Administration through the Arizona Department of Transportation.

#### Nature of and Approach to Problem

Frequently the soils available for construction cannot meet the requirements, such as strength and incompressibility, imposed by their use in embankments or subgrades. The process of improving the soil so that it can meet the requirements is known as stabilization.

Many different methods of stabilization have been proposed. From the standpoint of their function or effect on the soil they can be classified as follows:

1. Moisture-holding: Retain moisture in soil.
2. Moisture-resisting: Prevent moisture from entering soil or from affecting clay materials.
3. Cementing: Binding the particles together without their alteration.

4. Void-filling: Plugging the Voids.
5. Mechanical Stabilization: Improving the soil gradation.
6. Diagenesis or Physiochemical Alteration: Changing the clay mineral or clay mineral absorbed-water system.

A satisfactory stabilizing technique must provide the required soil qualities and in addition must satisfy the following criteria: (1) Compatible with the soil material; (2) Permanent; (3) Easily handled and Implemented; (4) Low cost.

The problem of stabilizing Chinle Clay has been under consideration by A.D.O.T. since the early 1960's because of the vast deposits of Swelling Clay in Northern Arizona, some 6,000 sq. miles of surface exposure, as shown in figure 1.

The swelling characteristics of these Chinle Clay deposits are large and have caused excessive heaving in highway subgrades. This volumetric strain with resulting pavement stress has caused damage or failure to many miles of existing highways in this region of Northern Arizona.

Recognizing this condition the Arizona Department of Transportation and the F.H.W.A. supported the work of Dr. Charles E. O'Bannon, of A.S.U.'s Civil Engineering Department, on diagenesis of Chinle Clay by electrochemical methods.

# MAP OF ARIZONA SHOWING MAJOR CHINLE OUTCROPS

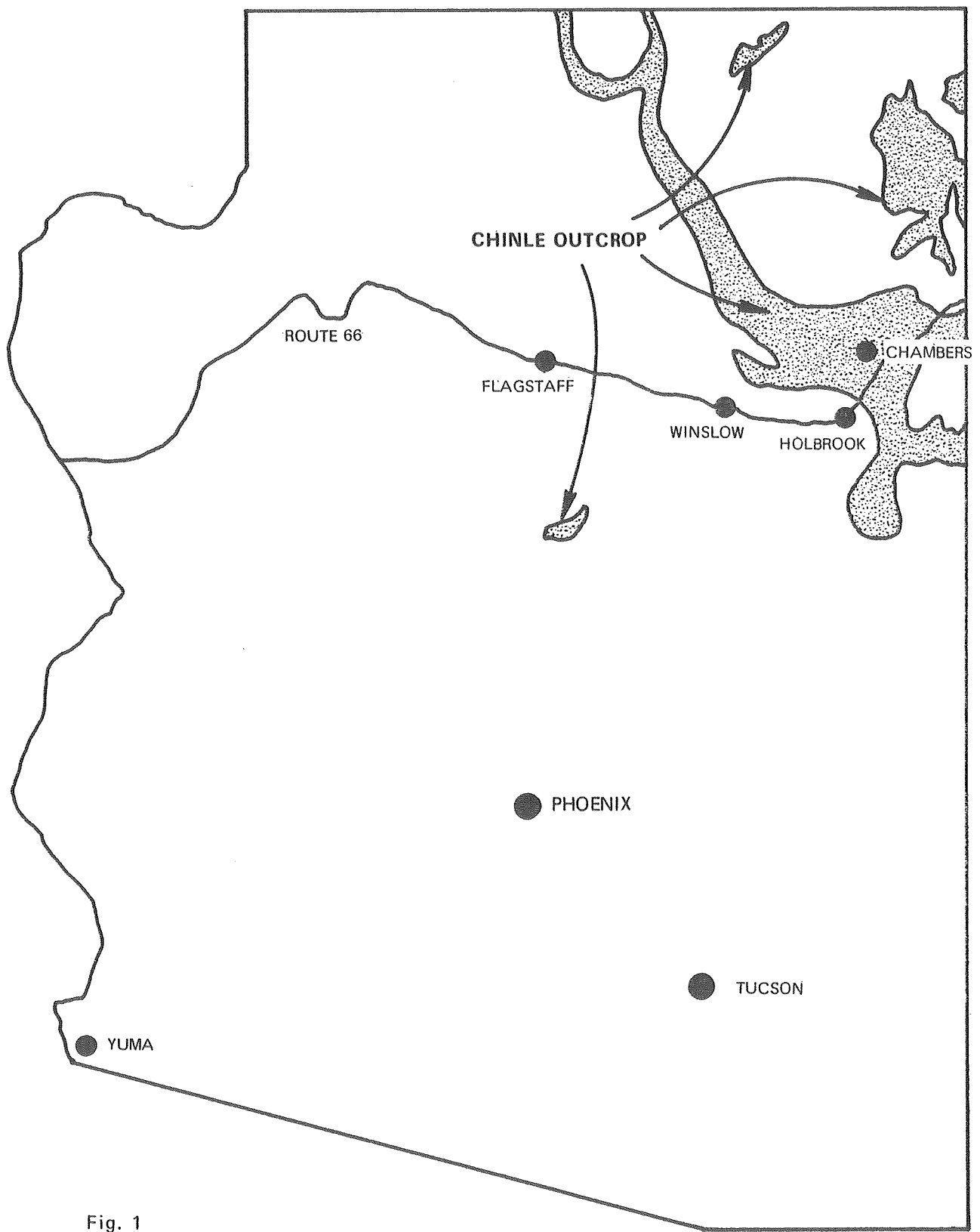


Fig. 1

## Object of Research

The object of this research was to utilize Dr. Charles O'Bannon's electrochemical soil-treatment technology and, if necessary to modify portions of it for the express purpose of creating a viable soil stabilization technique that is implementable by Highway Division Maintenance personnel.

## Scope of Report

This report on the Electro-Chemical hardening of Chinle Clay is presented in eight chapters. These chapters explain in detail the various areas of laboratory and field experimentation conducted since July 1, 1973. Chapter II discusses the Geology of site and the preliminary sampling layout. Chapter III is a review of some Electro-Chemical soil hardening concepts as it applies to the existing problem. Chapter IV presents the work conducted in the laboratory in preparation for the field test. Chapter V discusses the work performed in the field test in the spring and summer of 1974. Chapter VI presents the results of the field test with an analysis of the data. Chapter VII is a preliminary cost study for the method. Chapter VIII discusses the on-site moisture content measurements using Nuclear Gauges. Chapter IX is the conclusion and recommendation that have been reached during this study.

## CHAPTER II

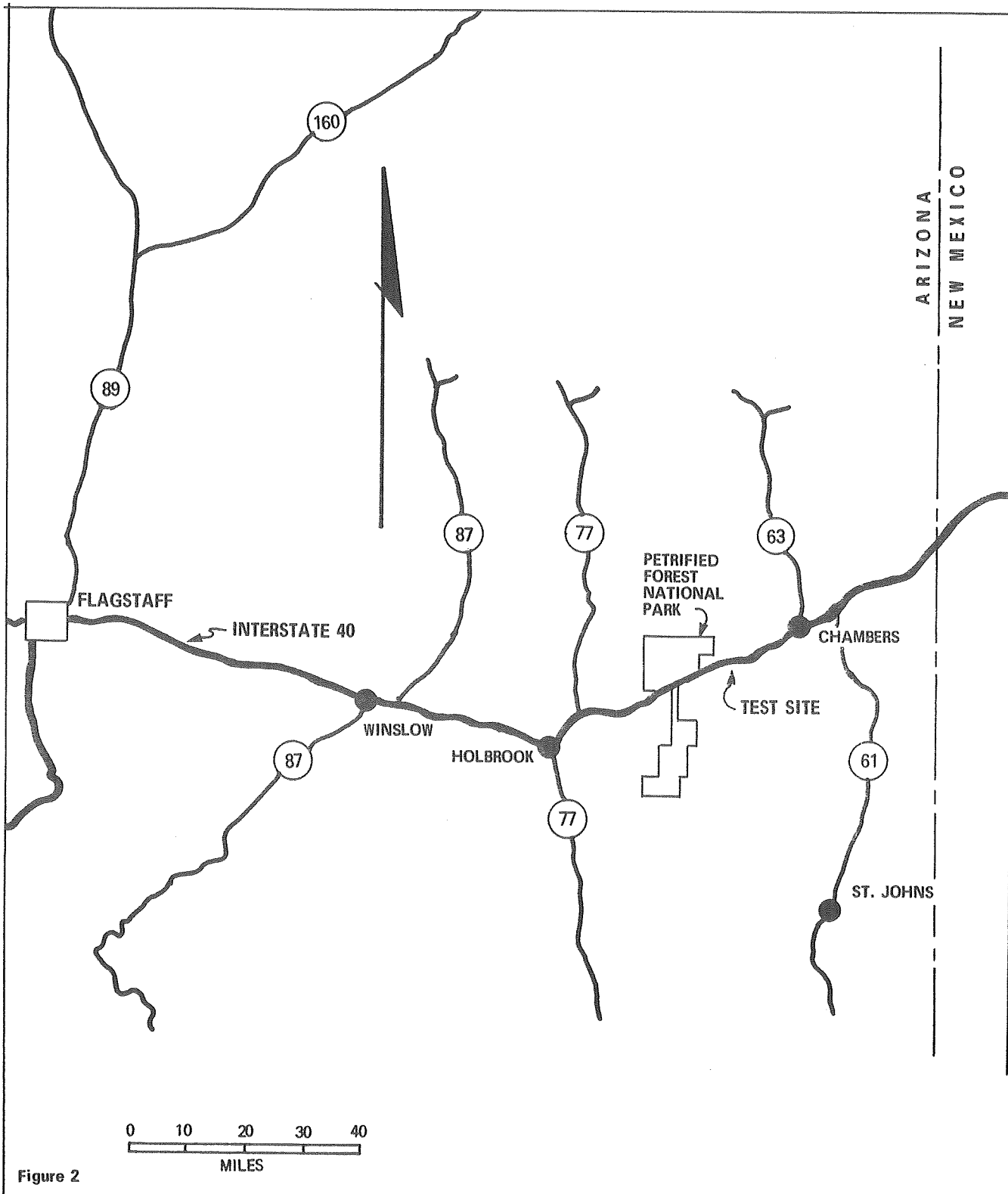
### GEOLOGY AND PRELIMINARY SAMPLING OF THE AREA AND SITE

The site is located in Northeastern Arizona on the Colorado Plateau geological province, see figure 2. The nearly horizontal sandstones and shales of Triassic and Jurassic age in this area are intermittantly covered by thin-bedded Tertiary sandstones and shales. This region is characterized by low, broad mesas overlooking wide, flat, stream valleys containing Quaternary to Recent sands and silt alluvium with low terrances containing a very limited quantity of gravel.

U.S. Interstate 40 transverses primarily the Triassic Chinle formation as it crosses this area. The Chinle formation is composed of shale, clay sandstone, and minor amounts of limestone.

Characteristics of the Chinle formation vary greatly in both the physical and chemical aspects. Chinle formation colors include red, pink, brown green, purple, and grey tints. It has been previously mentioned that shale, clay, sandstone, and minor amounts of limestone are present. Physical and chemical characteristics of sedimentary deposits are dependent for the most part on three factors: 1) Of course, dependence on parent material, 2) method of transportation of sediments, and 3) changes after deposition. Several depositional environments are responsible for the characteristics of the Chinle formation. It is held that the conditions under which the





chinle was deposited were wholly continental--probably those of a well-graded but rather arid plain across which streams meandered and on which there were perhaps scattered lakes. Conglomerates of resistant materials transported from a great distance are scarce; hence, it is evident that stream gradients were low. Continental origin is evidenced by the presence of fresh-water fossils, land vertebrates and the trees of the petrified forest. Evidence is present for the existence of a warm marine environment in portions of the area in which the Chinle outcrops. Montmorillonoid and bentonitic clay minerals are suggestive of volcanic activity at the time of the warm marine climate. The alteration of volcanic ash in such an environment is favorable to the information of the aforementioned clays.

In the general area of the site, the Chinle varies in thickness and had a maximum depth of 1,500 feet at the test site on Interstate 40, the highway is in a 15-foot shale cut. The weathered material on the slopes of the cuts extends to a depth of about one foot. This material has the typical reticulated appearance of an expansive clay, weathered in place from the parent material. The underlying material is very hard and brittle. The out-crop is fractured, slickensided, and shows no visible bedding planes.

During the first weeks of July, 1973 A.D.O.T. District IV personnel set down engineering reference stations on the west bound lane of U.S. Interstate 40 at M.P. 323.8± for approximately 1200' of the cut.

After due consideration with an all-system go target date of September 4, 1973 in mind it was decided that the optimal pre-sampling procedure, both from an engineering and economic viewpoint, was to acquire all soil samples in a continuous straight line pattern. The samples would be taken only over a 500 foot section of the cut selected as the test site.

Taken in this manner the soil samples would provide adequate data on the soil characteristics of the section for the purposes of this study.

It should be pointed out that this sampling procedure was conceived under the constraints of minimal time and money expenditures which would be essential considerations for an implementable maintenance operation.

However, this type of sampling procedure is not recommended if the soil study is to be one of a general investigatory nature.

#### Preliminary Sampling

Samples were obtained from sta 7+00 to sta 12+00 on 20' intervals and tested for expansive characteristics and atterberg limits.

Based on the atterberg limits the soil specimens were classified via the Unified Soil Classifications Systems. The result of the classification and a soil profile are shown in figure 3.

# SOIL CLASSIFICATION FOR I-40 E.O. SITE

MP 323.8±

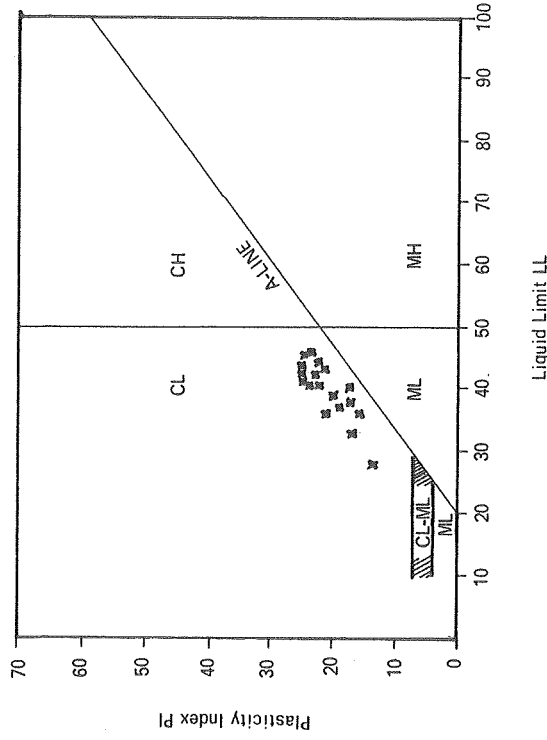


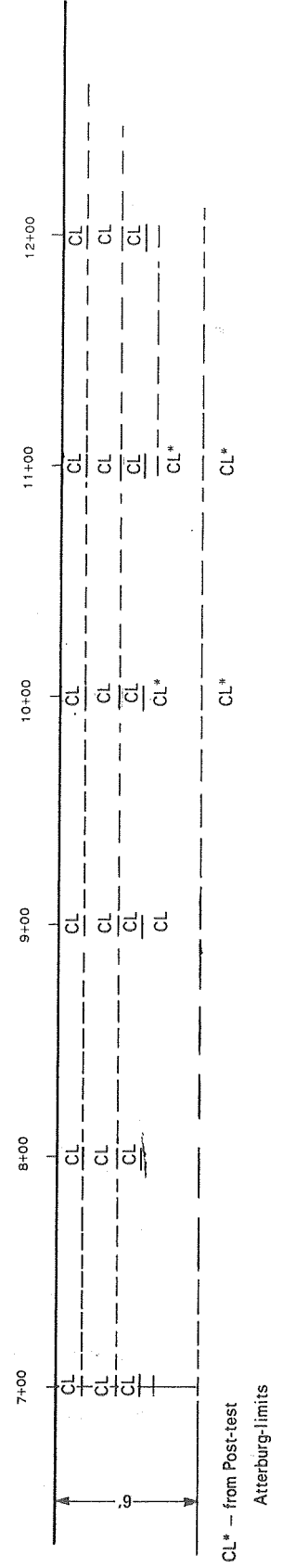
Fig 3

## CRAZY CREEK

### TEST - SECTION SOIL

#### CLASSIFICATION

SOME LENSES OF SANDY CLAYS, FINE SILTY SANDS  
CLAYEY SANDS, PRIMARILY MEDIUM PLASTIC CLAY



## CHAPTER III

### Physicochemical Aspects

#### Clay Structure

Because this work with electro-osmosis is primarily concerned with stabilization of Chinle Clay formations, a brief review of certain facts and models of clay minerology and electro-osmosis is fundamental.

It is well known that the soil particle size strongly affects the appearance and behavior of soils. These effects on soil behavior are due to the increasing magnitude of the forces among the molecules of adjacent particle surfaces as the size decreases. In silt or sand and larger soil sizes, the ratio of the area of the surface to the volume of the sample is relatively small, hence the intersurfaces forces small. In such cases the shape of the grain, surface roughness and resulting interparticle abrasive forces determine the overall behavior of the soil mass.

In the case of small soil particles, the molecules forming the particle surface constitute a large proportion of the total number of molecules, and the interparticle forces associated with these surface molecules have a significant effect on the behavior of the particle and hence, the mass of the soil.

It is found, in general, that allowance for the effect of surface forces must be made when the size of the particle becomes less than one micron ( $10^{-4}$  cm), a size which corresponds to the upper limit of colloids.

It was formerly believed that clay minerals were amorphous in structure, but investigations carried out with x-ray diffraction and electron microscopy since the 1920's have shown them to be predominately crystalline; moreover, it is found that clay minerals are crystalline hydrated aluminum silicates. Although the molecular structures are complicated, it has been shown that clay minerals are constructed essentially from two basic structural units: The silica tetrahedron ( $\text{SiO}_2$ ) and the octahedral aluminum hydroxide  $\text{Al}(\text{OH})_3$ .

A silica tetrahedron consists of a central silicon atom surrounded by four oxygen atoms arranged at the apexes of equilateral triangles. The other structural element, hydrated aluminum, takes the form of an octahedral crystal, in which the aluminum atom occupies the center of the structure, above and below which the oxygen and hydroxyl ions are arranged.

The clay minerals designated Smectites are composed of basic units of two silica tetrahedral sheets with a central alumina octahedral sheet. These units are continuous in the a and b directions and are stacked one above the other in the c direction. These basic units are referred to as silica-alumina-silica units.

In the stacking of the basic units, one above the other, the O-layers of each unit are adjacent to O-layers of the neighboring units. Because of the proximity of the O-layers, there is a weak bonding energy which permits easy expansion of the lattice in the c-direction. It is this high swelling characteristic that is the outstanding feature of Smectites.

Chinle Clay is a random-mixed layered mineral containing minor to major amounts of the Smectite mineral; i.e., about 10% - 40% by wt. of Smectite.

#### Base Exchange

Since Dr. O'Bannon's treatment of swelling clays rests on base exchange phenomena, a short discussion of this phenomena is in order.

The absorbed ions on a clay surface are present in a diffuse double layer. Each of the ions required to neutralize the charge on the particle surface is oscillating due to Brownian motion, thus the ion is assumed to be oscillating in a cell, called an oscillation cell, adjacent to a charged area on the particle surface. Other ions from an added electrolyte may enter the oscillation cells or may remain in the external phase. A given ion with a large naked radius, i.e.,  $K^+$ , will have a smaller hydration radius and thus be able to approach the charged surface closer than an ion with a smaller naked radius, i.e.,  $Na^+$ , and a corresponding larger hydration radius. Thus, the potassium ion will, on the average, bond to the surface with a correspondingly greater energy than a sodium ion.

Such concepts have been evolved into mathematical models which depict the ion exchange process as a stochastic process.

These theories, in particular the one by H. Jenny, lead to mass-law or mass-action equilibrium equations. However, these models of base-exchange imply the exchange phenomena is essentially a complex redistribution of ions

both between an external phase and the ion swarm and also within the ion swarm. Unfortunately this process depends on several factors. The ion redistribution cannot be regarded as simple metathesis, precisely defined by a simple equation of the mass-law type. The mass-law equations often quoted in the literature, with regard to base exchange phenomena, must be considered as only approximations from which there may be considerable variation in "unfavorable" cases.

Because of the stochastic nature of base-exchange phenomena, any soil treatment based on this phenomena must therefore also be regarded as a stochastic process. Of course one may raise the "odds" in favor of a successful treatment along these lines by preparing the external phase to be "rich" in the preferred ions, e.g., utilize a high concentration KCl solution to treat the soil.

It must be borne in mind that an ion-exchange soil treatment can alter the physicochemical properties of the swelling clay and is one of the most effective ways to combat problem over a relatively small localized region on which an expensive inplace structure, say a pavement, rests.

The ability of a clay to absorb ions on its surfaces or edges is called its base or cation (anion) exchange capacity, which is a function of the surface chemistry of the clay and the size of the clay particles. Thus the term base exchange is widely employed, although in fact hydrogen ions and even organic ions may be involved in the exchange.



The base exchange capacity of a clay mineral is related both to the charge deficiency caused by isomorphous substitutions within the lattice of the mineral and also to the number of dangling bonds around the edges of particles. The absorbed ion may be  $N_a^+ + C_a^{++}$ ,  $K^+$ ,  $Li^+$ ,  $F_e^{++}$  or other cations. These absorbed ions will not alter the basic clay chemical formula, but they significantly alter the engineering behavior of the clay mineral since they affect the bonding energy of the various basic structural sheets to one another. Moreover, in occupying spaces on the surface of the clay particles the ions interfere with development of water layers at the surface.

After analyzing this base-exchange phenomena, Dr. Charles O'Bannon of the A.S.U. Civil Engineering Department employed electro-chemical stabilization to the problem of swellings clays. Dr. O'Bannon felt that two primary effects could be relied upon to alter the clay properties.

- (a) The release of ions from the anodes, an exchange of ions in the soil solution and the migration of solutes in the pore water or in the absorbed water layers. The activation of such a system can lead to new particle configurations (e.g., closing or opening the space between clay platelets and locking them together with different cations either face to face or edge to face.
- (b) Mass-transfer phenomena, i.e., ionic transport and water transport to assist stabilizing solutions to penetrate into soils which may be otherwise nearly impermeable to the passage of solution.

In order to further illuminate the rationale behind Dr. O'Bannon's usage of electrochemical stabilization a short discussion of some fundamental ideas relating to swelling clays and effects of electrolytes on clay swelling is presented.

#### Classical Models of Swelling Mechanisms

The observed swelling pressure of montomorillonite is, of course, the difference between the attractive and repulsive forces operating between the layers per unit area.

At small interlayer distances two attractive forces predominate; namely, electrostatic forces between the negatively charged surfaces and the intervening exchangeable cations, and van der Waals forces. The magnitude of the former depends on the surface charge density resulting from isomorphous substitution. Thus, the forces between the charge surface and the cations depend on the composition of the mineral. The van der Waals forces are not greatly dependent of the material composition; however, they are influenced by the external environment.

The repulsive forces involved have been generally considered to be osmotic in nature. It is assumed that when montomorillonite is placed in contact with water, the water initially enters between the layers because the molar energy of the water is reduced by the hydration of the cations present between the layers and hydration of the layer surfaces.

Hence, a highly concentrated solution of cations is formed between the clay layers. Moreover, because of the negatively charged layers an electrostatic field is present which constrains the cationic thermal motion.

This thermal or Brownian motion of the cations is always present and without the constraint of the interlayer electrostatic field the cations between the layers would diffuse out to the exterior solution until an equilibrium concentration was established.

The osmotic model used to describe swelling phenomena is formulated around these facts as follows: Because the interlayer electric field acts as a semipermeable membrane and causes the interlayer solution to have a high concentration of cations, exterior water enters due to an osmotic gradient. The resulting repulsive pressures is, thusly, assumed to be an osmotic one.

This osmotic theory of swelling seems to be a plausible one and indeed some experimental evidence at least qualitatively, supports it. However, Dr. Phillip F. Low, among others, suggests that this is not the full story. It has been shown thermodynamically that the repulsive pressure has not only the osmotic contribution, but also a contribution from the molar potential energy of the water.

The molar potential energy is a factor because the interlayer water interacts with lattice structure of the clay crystal. That is the structures of the interlayer water and montmorillonite conform to one another, i.e., epitaxy occurs, because of the hydrogen bonding between them. What this means is that as the thickness of the interlayer water increases it's structure assumes a preferred configuration and the montmorillonite crystal adjusts

accordingly. This adjustment continues until the water achieves its preferred configuration. An alternative interpretation is that the montmorillonite structure relaxes as stresses acting on it are relieved by the inclusion of water, and the water structure changes conformably. This relaxation continues until the water structure resists further strain.

Now as this phenomena occurs the b-dimension of the unit cell\* increases due to an alternative clockwise and counterclockwise rotation of the clay's silica tetrahedra about their vertical axes. See figures 4 and 5. This rotation of the tetrahedra alters the b-dimension of the unit cell. Thus, all maximally swollen montmorillonites have the same b-dimension for their silica tetrahedra. This value is a little over  $9 \times 10^{-8}$  cm or  $9\text{\AA}$ .

It has been established that the presence of potassium ions in the interlayers of montmorillonite significantly influences the lattice structure of the clay. Work done by Dr. Phillip Low and others shows that the potassium ion increases the b-dimension, see figure 6, hence reduces the swell potential of the montmorillonite clay. Dr. O'Bannon has found that the required amount of potassium to significantly affect the swell of the Chinle Clay varies from about 0.008 gms K<sup>+</sup>/gm clay to about 0.02 gm K<sup>+</sup>/gm clay.

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\*The unit cell is the smallest structural unit which, when repeated in space, reproduces the given crystalline structure.

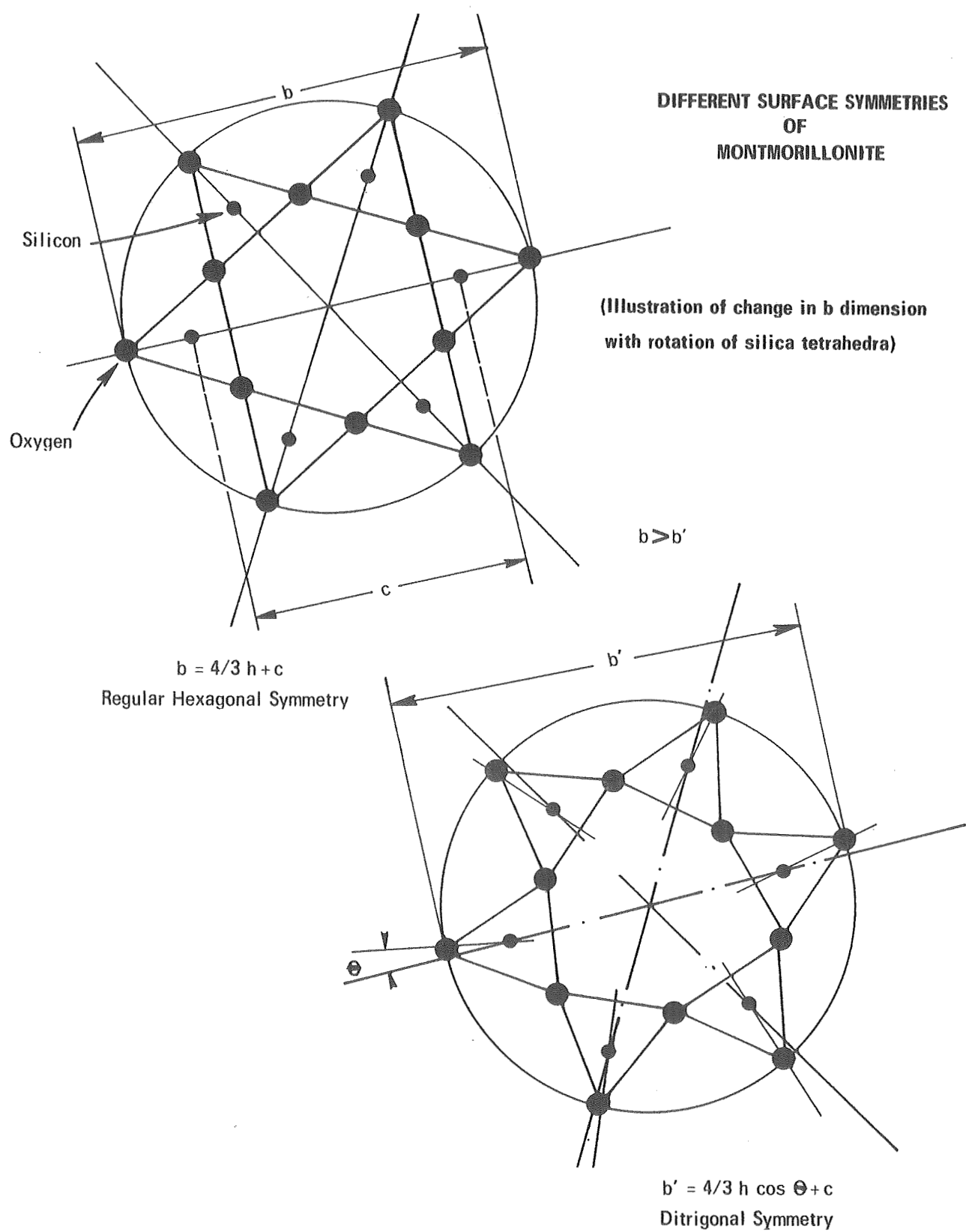
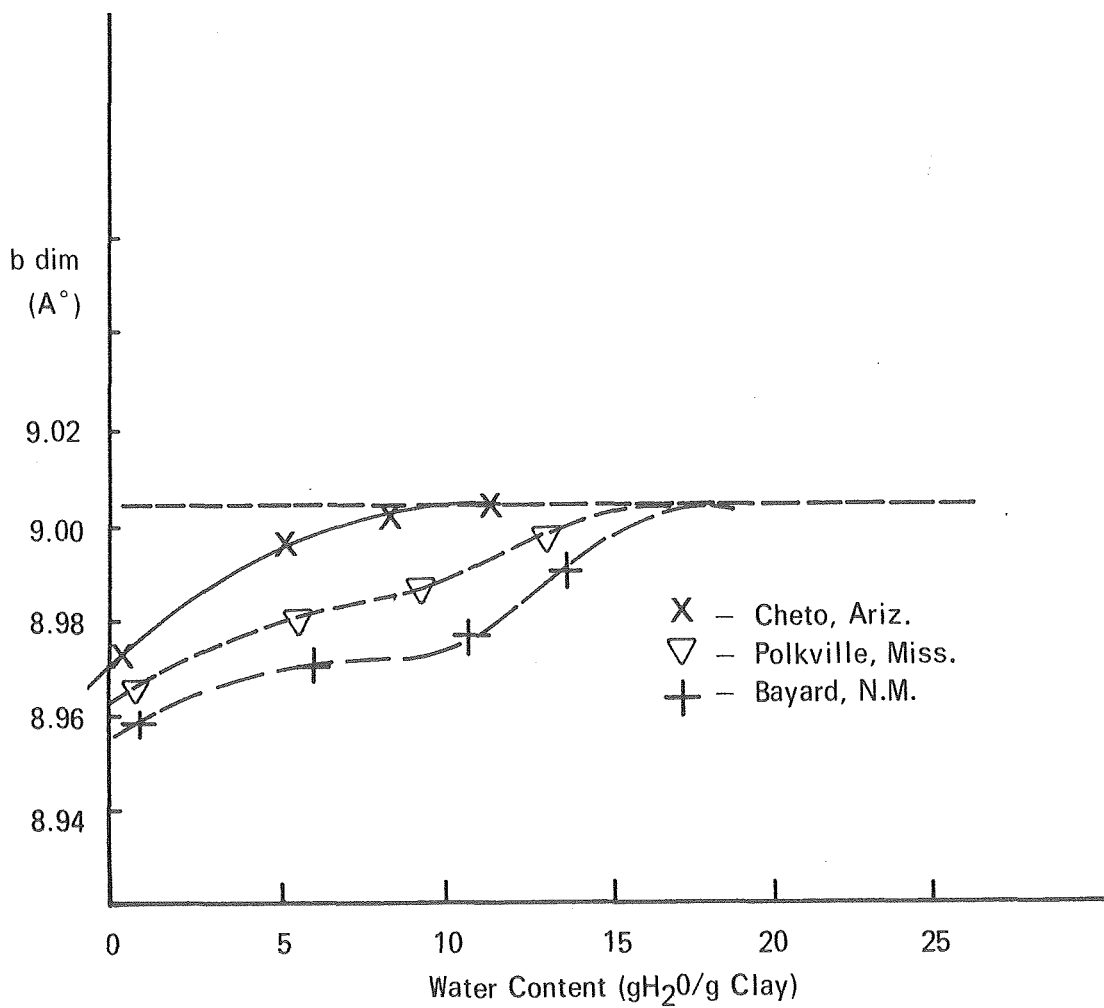


Fig. 4



Relation between b-dimension  
and  
Water content for N<sub>2</sub>-Saturated  
Montmorillonites (Ravina & Low)

Figure 5

THE EFFECT OF THE EXCHANGEABLE CATION  
ON THE B-DIMENSION OF NA-MONTMORILLONITE

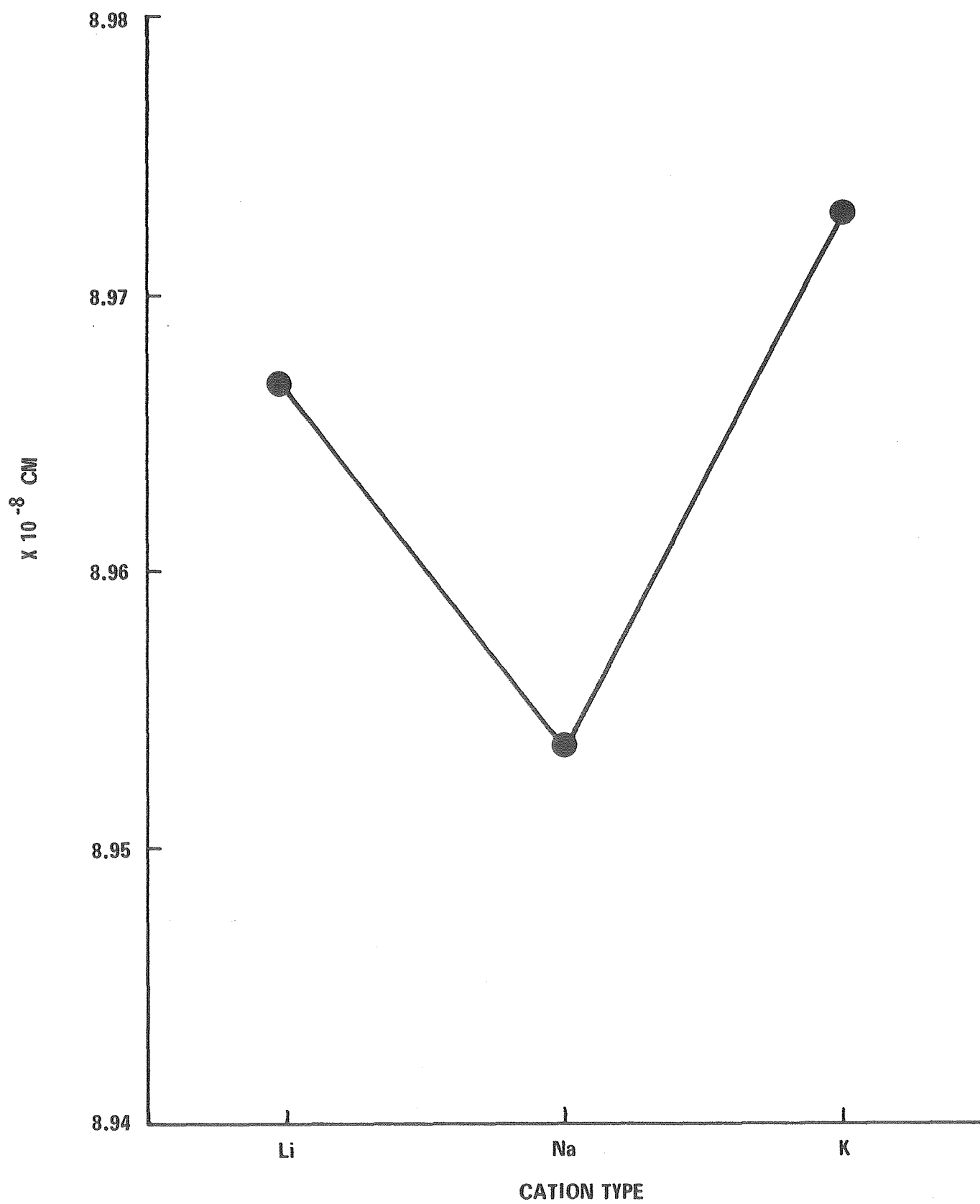


Fig. 6

## Clay Stabilization by Electro-Osmosis and Base Exchange of Ions

It is generally believed that Dr. O'Bannon utilized an electric field to move a chemical solution through the clay region solely because of the low hydraulic permeability coefficient of Chinle Clay. This, however, is not strictly correct. It is well known\*\* that electrolytes have an effect on the magnitude of the soil permeability. For example field & laboratory work for this project showed that a 0.4 N KCl solution moves through Chinle Clay, under a hydraulic gradient, at a significantly higher velocity than deionized water does (i.e., well over an order of magnitude increase in velocity).

Because of this, several attempts were made in previous studies, to introduce the KCl solution into the soil using only hydraulic gradients. Thus with a simple "ponding" technique one could rely on the base-exchange of ions to effect a change in the clay structure. While base exchange produces long-term effects on the clay structure it was found that application of an electric field induces additional electro-chemical phenomena which also greatly altered the soil structure.

With the application of the electric field to the KCl solution - saturated clay a variety of electro-chemical phenomena occur. Among the various mass-transfer effects are ion-transport and water-transport.

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\*\*Lutz, J.F., and W.D. Kemper, "Intrinsic Permeability of Clay as Affected by Clay-Water Interaction", Soil Science vol. 88, 1959, pg. 83.



O'Bannon's stabilization method relies primarily on ionic-transport to effectively move the potassium ions to the interstitial clay sites. However, in conjunction with the ionic-transport, water-transport occurs, and so O'Bannon's soil stabilization technique was initially categorized as employing electro-osmosis for the purpose of soil stabilization. Clearly, this is not the most comprehensive term to use for this process.

In fact as soil stabilization research work proceeded, it became clear that O'Bannon's electro-chemical technique can be characterized as basically the electrolysis of swelling clays.

## MISCIBLE DISPLACEMENT

### Introduction

Since the soil stabilization process as developed by Dr. O'Bannon consists of transporting KCl solution through the soil, it was found to be necessary to examine what conditions affected this mass transport and how they affected it.

The study of the movement of water, containing dissolved solutes, into and through the soil is called Miscible Displacement.

The term Miscible Displacement is a relatively new one, however, the technique is very old and has long been used. For example, reclaiming saline and alkali soils by leaching out the excess salts has been used for many years but the process has been examined analytically only recently.

### Solute Displacement

Briefly, the following takes place in a porous medium as one fluid, say pure water, displaces a KCl solution.

The solutes, in this case KCl, in the displaced fluid are transferred through the soil by the mass transport of the moving fluid and by thermal agitation. Fluid flow through soil is, of course, very complex. The fluid velocity fluctuates rapidly due to the complex geometry of the medium. This complex flow causes the KCl in the solution to move into the pure water. The

term for this mass-transport is dispersion, often called hydrodynamic dispersion, to distinguish this spreading mechanism from that due to diffusion. This distinction is necessary because diffusion is due to the random thermal motion of the solute molecules, whereas dispersion is due to the macroscopic fluctuations in the flow of the fluids through the complex pore systems. Thus, the movement of a solute across the initially sharp boundary between the displacing and the displaced fluid can be due to dispersion or diffusion or both.

The shape of a boundary between two different fluids and concentration gradients as they emerge from a soil column could yield information about the magnitude of viscous drag of the fluid on the pore walls, the velocity distribution of the fluid in the pores, the configuration of the pores, the amount of diffusion taking place in the pores, and even the degree of ion exchange taking place. However, such an experimental procedure is not easy to set up, in fact, little experimental work using such a procedure has even been attempted.

Another approach adopted by Drs. O'Bannon and Mancini for this project, is to examine the concentration change of a displaced solute in the effluent as the boundary of the displacing fluid and the displaced fluid emerges from a soil column. The manner in which the concentration changes can give us some information about the porous media and the physical behavior of the fluids moving through the media.

#### Experimental Procedure

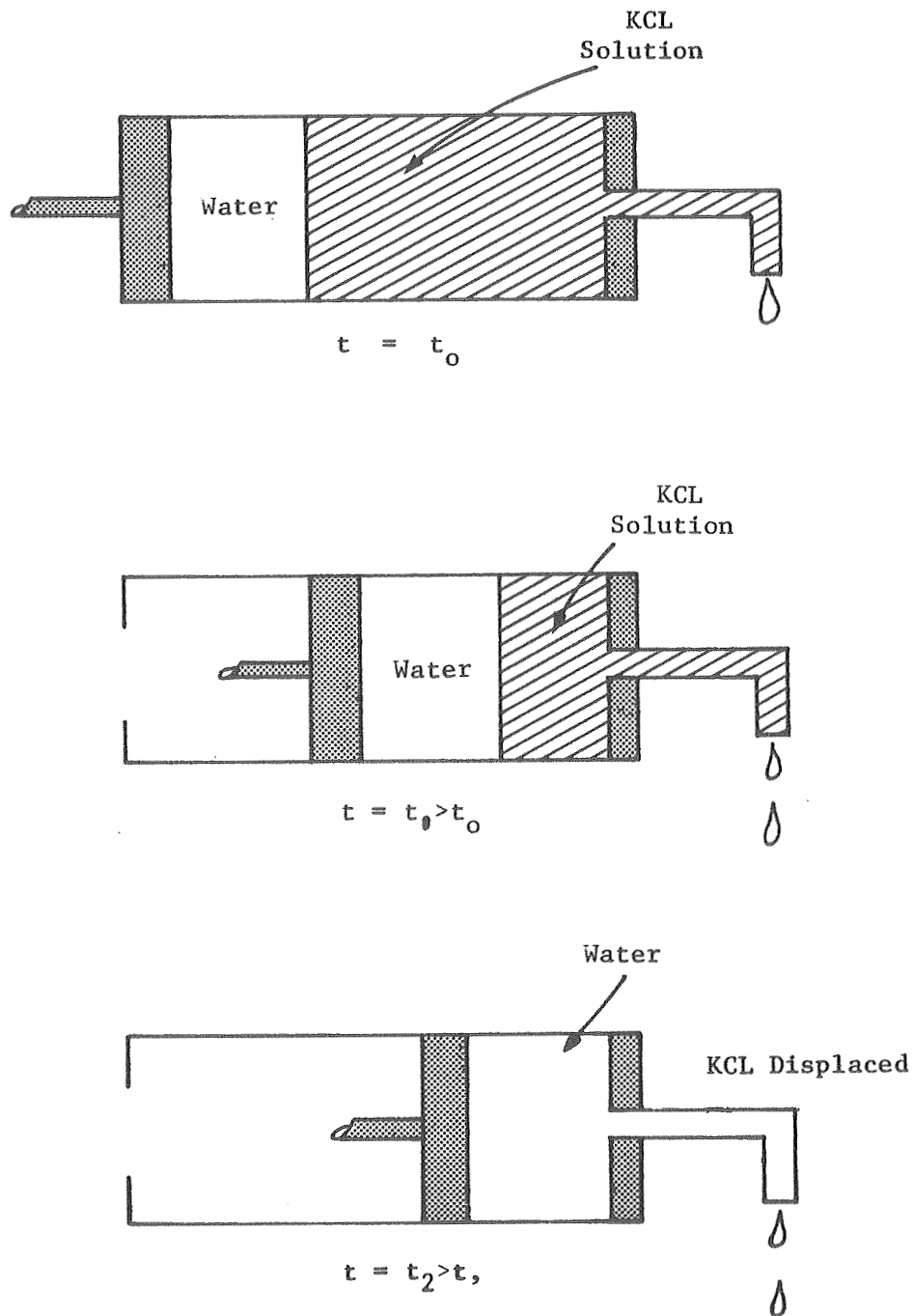
To analyze the concentration change of solute in the effluent from a soil column the data is presented in a standard form called a breakthrough curve.

A breakthrough curve is a graph of the ratio  $C/C_0$  versus the number of pore volumes of effluent collected. Here  $C$  is the concentration of the solute found in the effluent,  $C_0$  is the initial concentration of the solute in the displacing fluid and the pore volume  $V_0$  is the volume of the porous medium occupied by fluid. The number of pore volumes of effluent collected is given by  $Qt/V_0$  where  $Q$  is the quantity of flow per unit time and  $t$  the interval of time that has elapsed since the displacing fluid was added to the medium. The quantity  $Qt$  is also the volume of effluent collected. The ratio  $C/C_0$  will be zero at first and then approach 1 as the concentration of the effluent approaches that of the displacing fluid, that is when most of the displaced has been removed from the medium and the effluent is composed mostly of the displacing fluid.

There are two classes of Miscible Displacement, they are "piston flow" or flow without mixing and flow with mixing.

Piston flow is schematically shown in figure 7. In the upper part of that figure there is a tube with a piston at the left end and a small outlet tube at the right end. In the tube there is a salt solution, say a saturated KCl solution, initially separated from pure water by a membrane. If we pull out the membrane and push the piston fairly rapidly to the right so that there is no time for diffusion to occur across the boundary line between the salt solution and the water, the water will be moved to the right without any mixing at the interface. This is shown in the middle part of figure 7. The effluent will be KCl solution until the water front reaches the end of the tube. Then, only pure water will come out of the tube as shown in at the

# Miscible Displacement



Schematic : Drawing of Piston Flow in a Tube, no mixing involved.

bottom of figure 7. Hence, in this type of flow the displaced fluid will move at the rate of the moving piston. In piston flow there is no viscous drag of the fluid along the walls of the tube and no turbulence, moreover piston flow almost never occurs in soils.

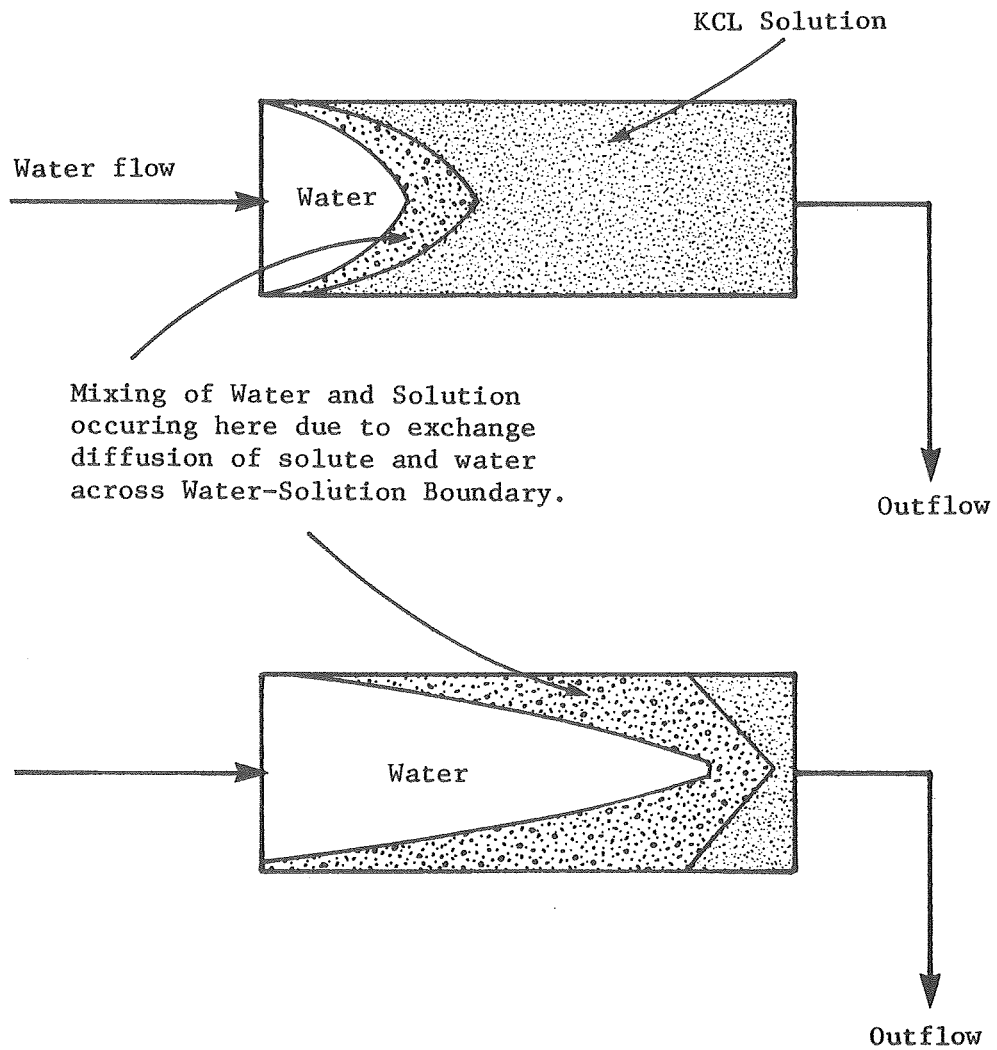
A more realistic type of flow through a tube is shown in figure 8-a. The tube is shown to exhibit a drag on the fluid. It is shown in the upper part of this figure that as the water is pushed into the tube at the left it moves faster down the center of the tube than it does along the walls. This is, of course, due to the viscous drag of the fluid near the wall which causes the velocity of the fluid next to the wall to be less than that near the center of the tube.

In the lower part of the figure it is shown that the salt front has advanced much further along the axis and it will not be long before the water will move out of the tube and be found in the effluent. Then as additional water is moved through the tube the concentration of the salt in the effluent will decrease. This is distinctive from the pure piston type flow since not just salt solution then water will move out of the tube, but an effluent with a measurably decreasing concentration of salt. It is shown in theoretical treatments that the smaller the diameter of the tube and the longer its length, the greater is the mixing.

#### Breakthrough Curves

An example of breakthrough curves for both piston flow and flow in a tube are provided in figure 8-b. Notice that for piston flow one pore volume of effluent passes out of the medium before any change in the concentration

## Miscible Displacement



Schematic drawing of Actual flow in tube

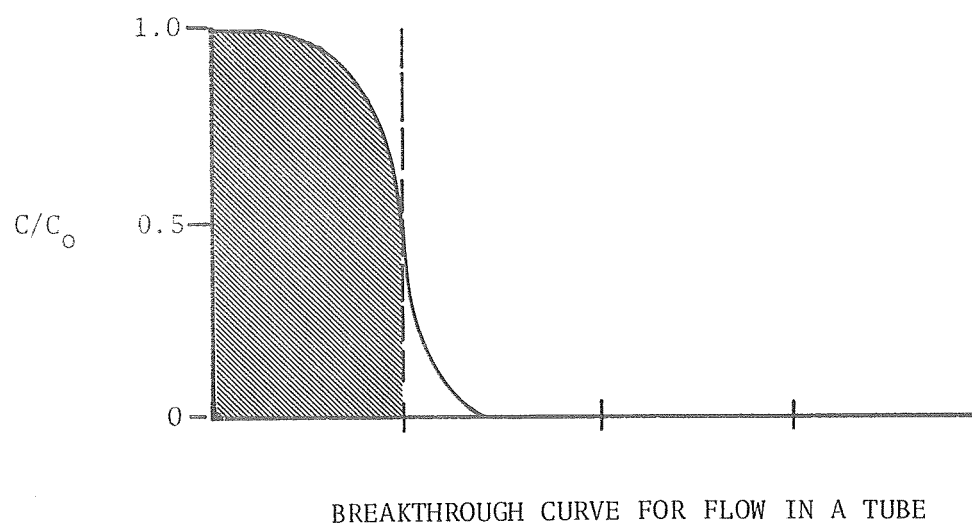
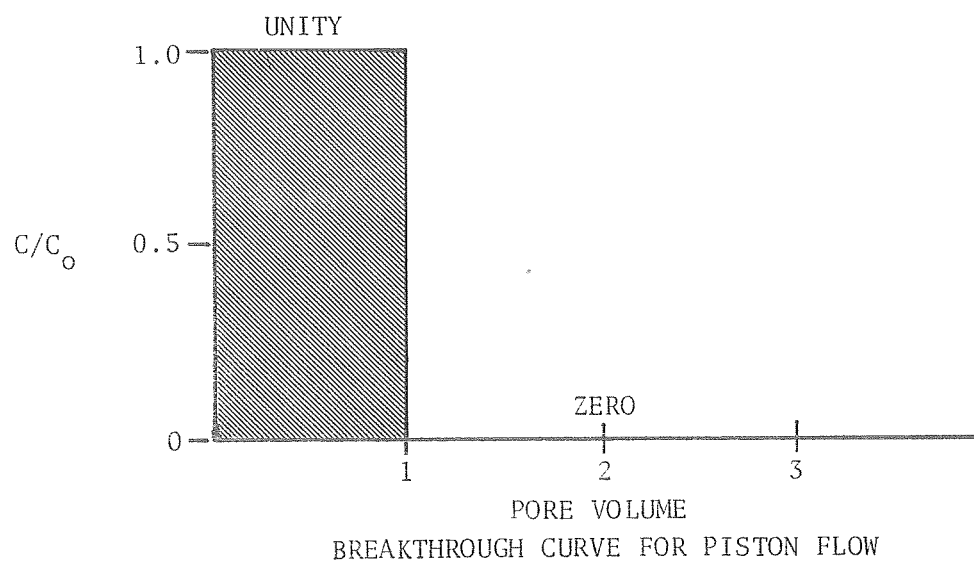


Figure 8-b



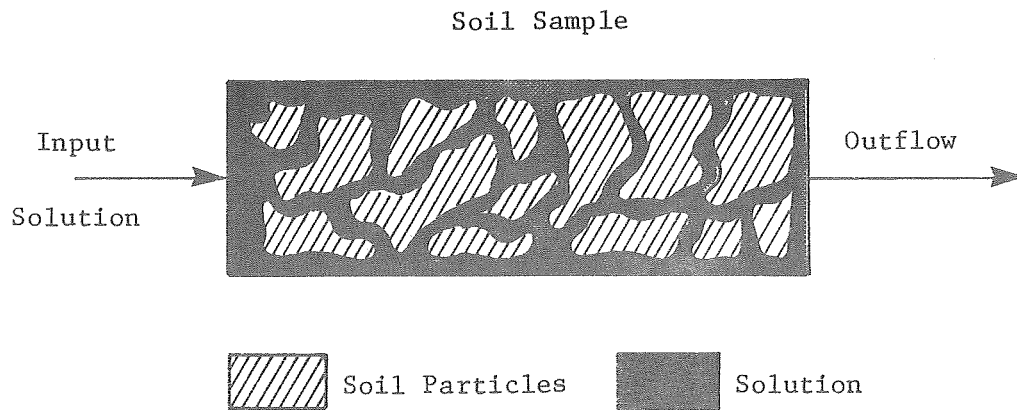
ratio  $C/C_0$  takes place. Then, in a very short time interval the concentration ratio goes to zero. In the case of flow through a tube the concentration ratio remains constant until about a pore volume or so is collected then the concentration ratio begins to decrease in a relatively gradual manner.

In figure 9 one sees how a displacing fluid will move through a saturated non-swelling soil. The displacing fluid will tend to move mostly through the large pores and displacement will be slow in the smaller pores, moreover diffusion will occur.

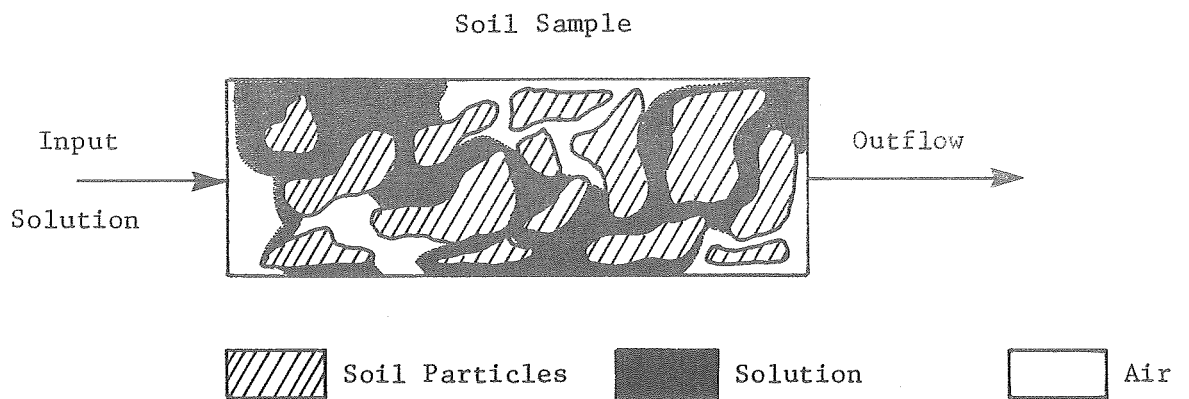
In laboratory experimental set-up the soil sample was not saturated, the moisture content being about 5 percent. Indeed it was found that if the soil was saturated with deionized water then the electrolyte would not move into the soil even under applied pressure gradients. With unsaturated soil the situation is more complex than for saturated soils. This case is shown in figure 10 where there are soil particles, water and air spaces. It should be stressed that this figure is drawn without taking into account any volumetric change due to swelling. In this situation the large pores, which, under saturated conditions conducted the salt solution easily, are now partly filled with air and have lost their conducting effectiveness. The salt solution will tend to go where the water films are the thickest.

Diffusion of a solute may take place during miscible displacement. If the fluids of the miscible displacement process move in the soil at practically zero velocity, some of the salt molecules may move forward and completely out of the soil just by the thermal motion of diffusion.

## Miscible Displacement



Schematic drawing of flow in a  
Saturated Soil



Schematic drawing of flow in an  
unsaturated Soil

Under field conditions one will almost always have an unsaturated soil medium at hand. In this case the theory as developed can only serve as general guides for qualitative judgments. This is due to the wide range of pore velocities present, during the displacement process, in the unsaturated soil.

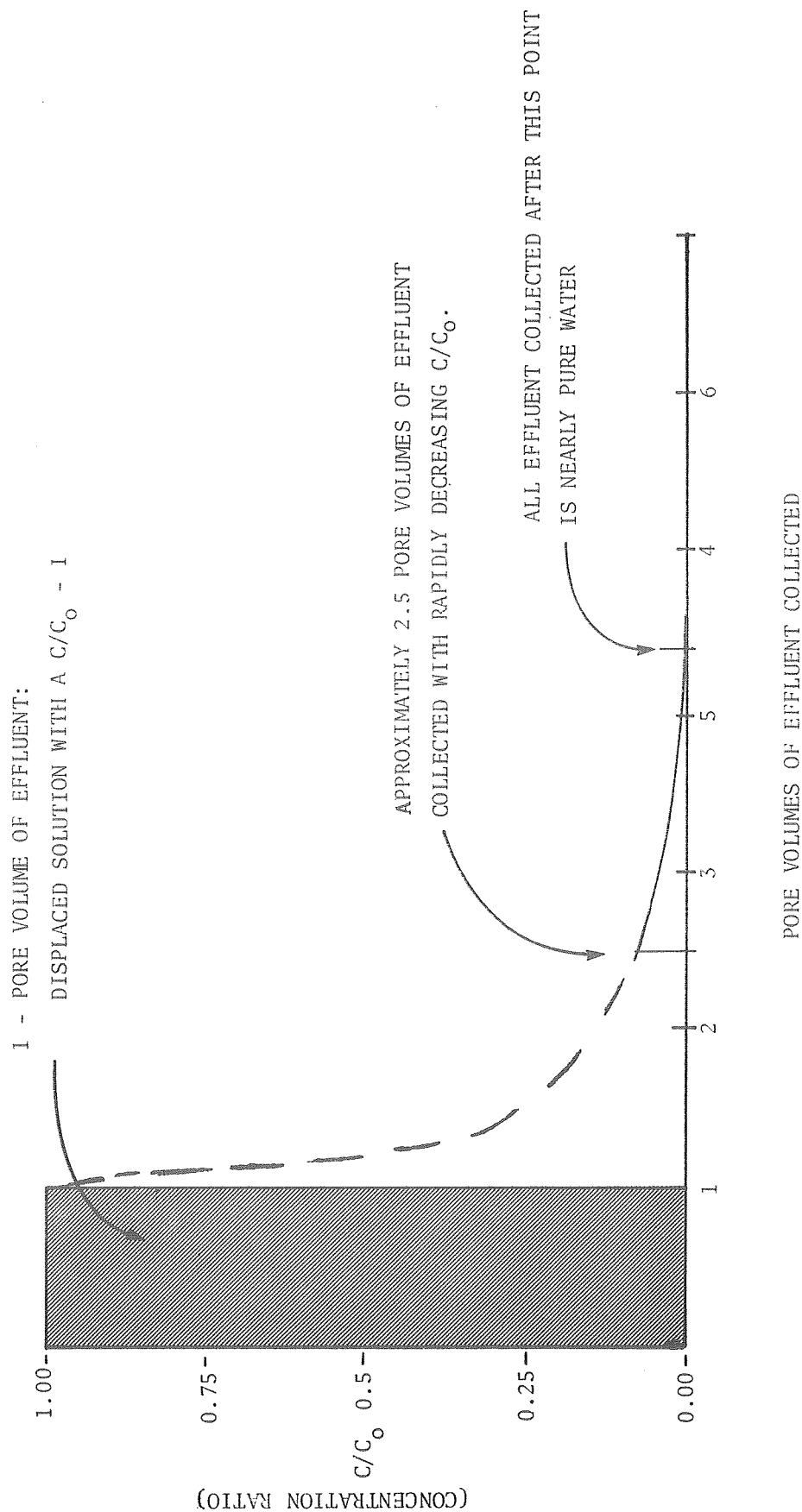
The miscible displacement experiments performed for this project were necessarily restricted in scope because of the limited time available.

It is apparent from figure 11 that after a Chinle Clay soil is saturated with an electrolyte (0.4 N KCl) and then the solution is displaced with deionized water the removal of the chlorides takes place very nearly like a classical "breakthrough" curve. This data suggests that the colloidal particles making up the soil have agglomerated, from the action of the electrolyte on the double layer, and created continuous pores so that the solution and water can pass through the clay column with relative ease.

More over even after several ml of deionized water were run through the sample and the effluent's salt concentration had reached an equilibrium value the clay's properties did not significantly alter and the sample retained its relatively high permeability relative to the deionized water.

In attempting to run the experiment in the other direction, that is by using a sample saturated with deionized water and then using the electrolyte as the displacing fluid, our efforts were thwarted by the properties of the clay. That is, the clay so tenaciously held onto the deionized water that the 0.4 N KCl solution was not able to enter the soil and displace the water, this occurred even with applied pressure gradients.

MISCIBLE DISPLACEMENT OF KCL SOLUTION  
FROM  
CHINLE CLAY SOIL COLUMN



These experiments clearly indicate that the electrolyte strongly interacts with dry Chinle Clay and is able to easily affect the permeability of the clay if clay has not been pre-saturated with pure water.

## CHAPTER IV

### Laboratory Work

#### General

The purpose of the laboratory work during Phase VI was to determine the expansive and swell characteristics of the untreated soil obtained from the test site.

#### Site Selection and Sampling

The field work for this study began in July, 1973, with the selection of the test site at mile post 328.8± on the westbound lane of Interstate 40. For a plan view of the site see figure 12. Since this project was initiated as an implementation study an engineering decision as to pre-test sampling technique was made at the onset of the project.

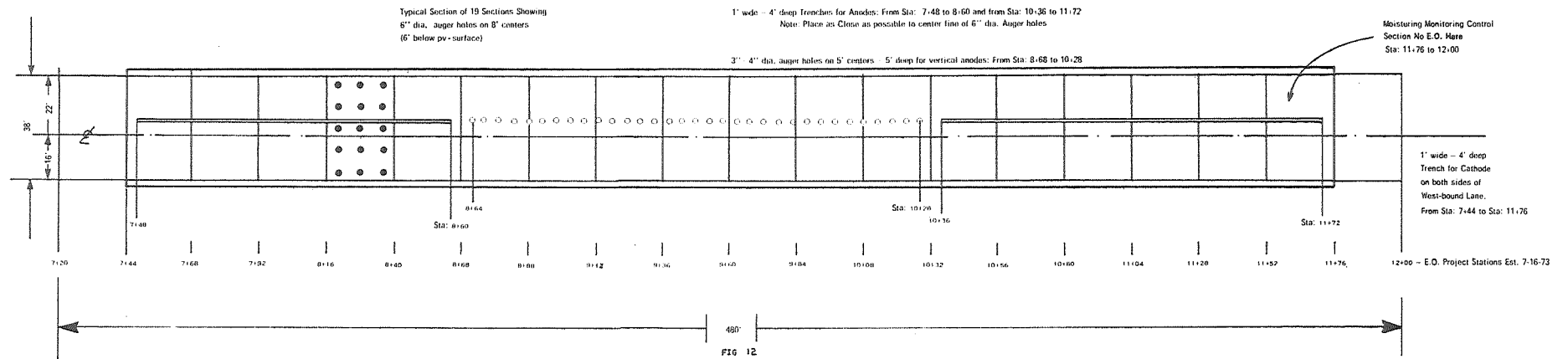
This engineering decision is based on the following general considerations: most problems areas of swelling clays traversed by a roadway are easily identifiable visually. After an area has been indentified as a possible problem area, a quick economical way of sampling must be used. It is clear that in a study oriented towards purely investigatory goals of soil phenomena a sampling technique used over a selected region should be based on a random procedure. However, the primary goal was to utilize knowledge of past electrochemical work and apply the same techniques, properly modified, to enable a rapid evaluation of the swelling problems of a given area.



LAYOUT OF TEST SITE FOR SOIL STABILIZATION PROJECT

LOCATION: I-40 WESTBOUND LANE

(CRAZY CREEK)



It was decided that the sampling procedure would be undertaken so that the drill rig would not have to move laterally but only longitudinally, this would save a great deal of time, hence money. Moreover, the rig was positioned as close to the center of the interstate as possible without interfering with traffic in one travel lane of the westbound roadway. Twenty-six test holes were drilled and samples were obtained to a depth from zero to 15 feet in the clay subgrade. See figure 13. The samples were transported to the Arizona Department of Transportation Materials Services where they were prepared and used in laboratory studies.

#### Sample Preparation

The samples were prepared in the following manner, see figure 14. The soil was sieved through a #4 sieve. All material passing the #4 sieve was stockpiled. The material retained on the #4 was crushed in a jaw crusher. The material was again sieved and the minus #4 added to the stockpile. This crushing and sieving process was repeated until all the material had passed the #4 sieve. The minus #4 material was then mixed and stored until needed for laboratory testing. In this manner, a homogeneous sample could be obtained for all further testing operations. See figure 14 for the flow diagram of laboratory work.



Note Stations 0 + 00 - 13 + 40 were set down July, 1973 by Dist. IV for Project HPR-1-11 (145)

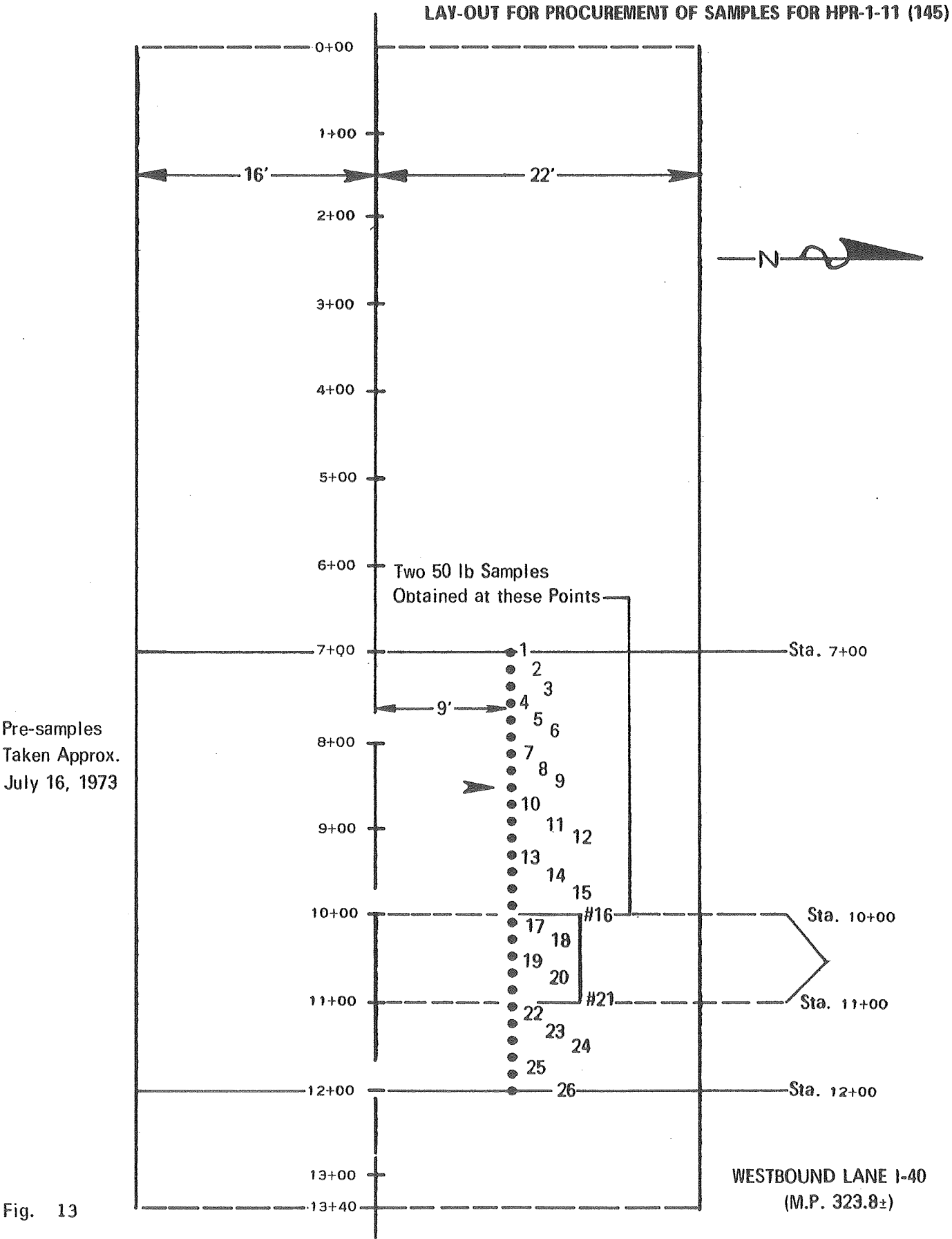


Fig. 13

# FLOW CHART FOR EXPANSIVE PRESSURE

(% SWELL FOR UNTREATED SAMPLES)

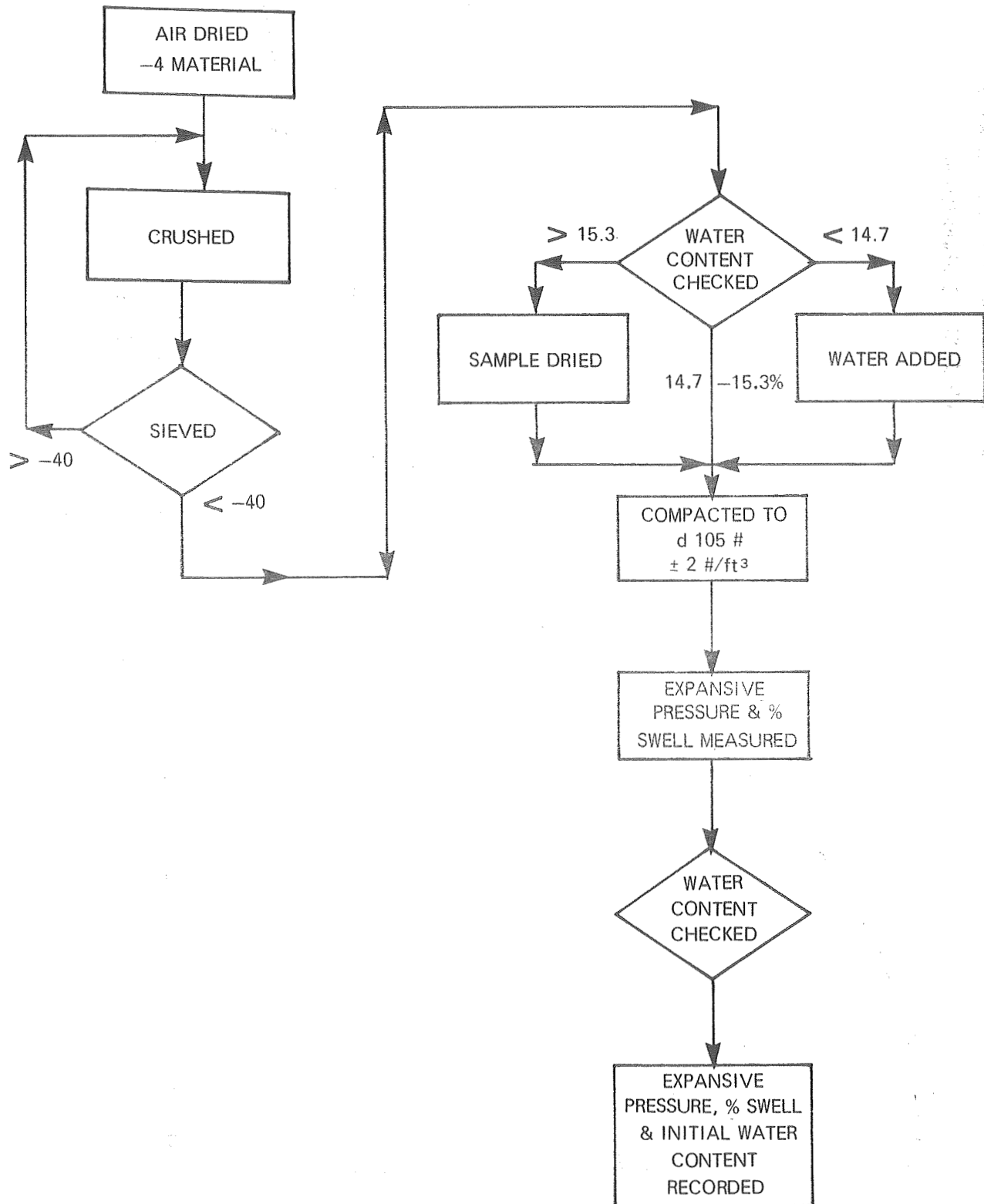
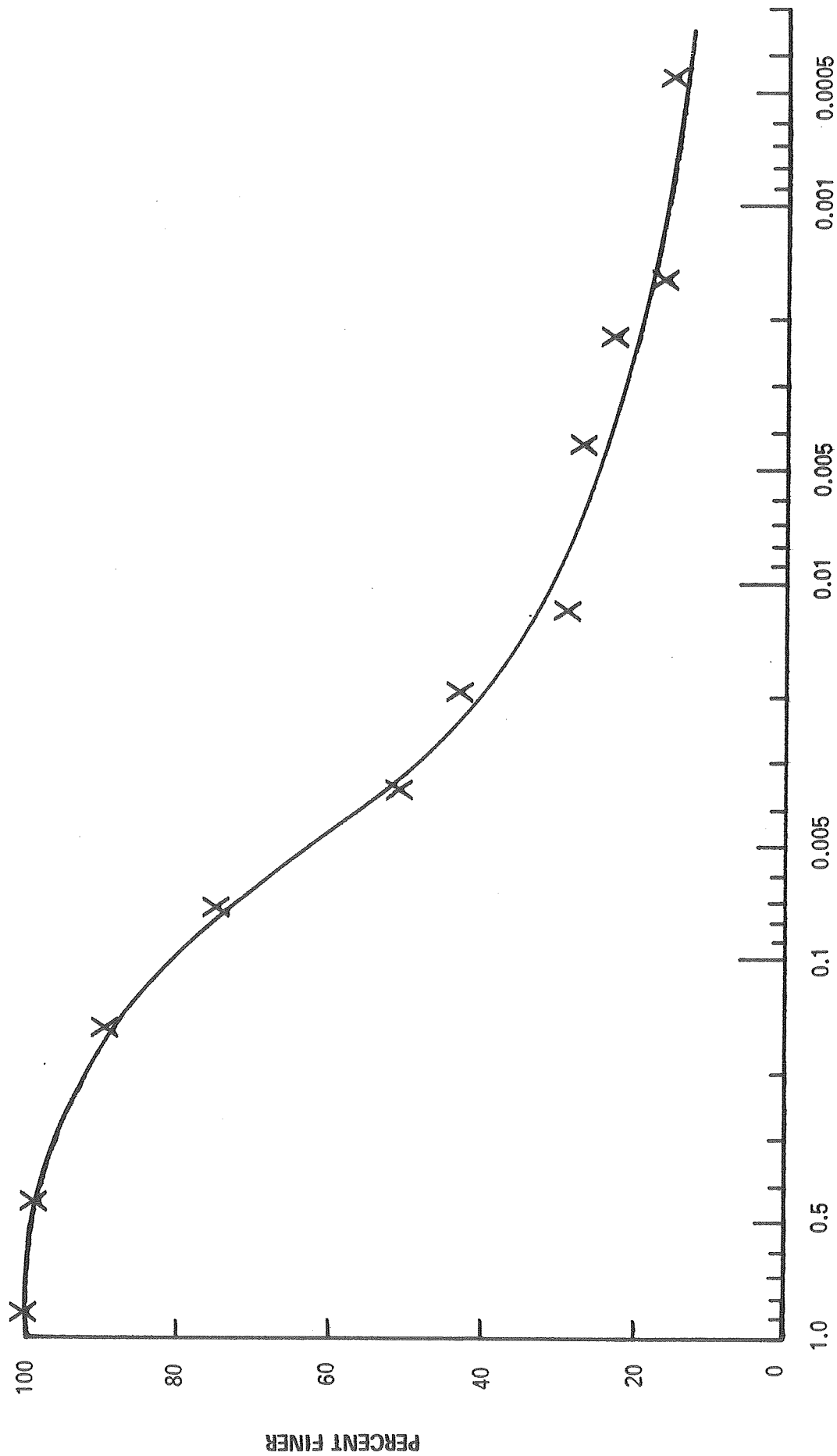


Figure 14

I-40 ELECTRO CHEMICAL SOIL STABILIZATION SITE  
GRAIN SIZE DISTRIBUTION



GRAIN SIZE IN (MM)

Fig. 15

### Soil Properties of Untreated Material

A series of tests were conducted to determine the index properties and grain size distribution of the soil. The average values for the atterberg limits on the samples were liquid limit 39, plastic limit 17, and plastic index 22. The average percent passing the #200 sieve was 70% and its grain size distribution is shown in figure 15. The specific gravity of the soil was 2.75. The soil would classify as a CL material based on the unified soil classification system. See table No. 1 and figure 3 for the results of the atterberg limits tests.

### Expansive Pressure and Percent Swell Tests

The expansive pressure of the untreated soil was determine for material passing a #40 sieve in a standard R-value testing apparatus. The density of the soil was approximately  $106 \text{ #/ft}^3$  and the moisture contents were 10 and 15 percent. The results of these tests are shown in table No. 2. The percent swell was determined on the untreated soil using a modified clock house apparatus. The density was maintained at  $106 \text{ #/ft}^3$  and the moisture contents were approximately 10 and 15 percent. The results of these tests are also shown in table No. 2.

These tests indicate that the selected area is representative of a region with moderate swelling characteristics.

Table No. 1  
INDEX PROPERTIES

Sample No.	Location	Depth ft.	LL	PL	PI	%-#200	Unified Classification Symbol
40133	7+00	0-1.5	43	22	21	92	CL
40134	7+00	1-5-2.5	37	18	19	78	CL
40135	7+00	2.5-3.5	35	17	18	60	CL
40149	8+00	0-1.5	42	16	26	76	CL
40150	8+00	1.5-2.5	40	15	25	60	CL
40151	8+00	2.5-3.5	43	17	26	76	CL
40165	9+00	0-1.0	28	14	14	48	SC
40166	9+00	1.0-2.0	36	14	22	66	CL
40167	9+00	2.0-3.0	32	15	17	61	CL
40181	10+00	0-1.0	37	19	18	75	CL
40182	10+00	1.0-2.0	42	19	23	83	CL
40183	10+00	2.0-3.0	40	17	23	80	CL
40197	11+00	0-1.5	38	17	21	65	CL
40198	11+00	1.5-2.5	44	19	25	66	CL
40199	11+00	2.5-3.5	44	20	24	77	CL
40213	12+00	0-1.5	36	16	20	57	CL
40214	12+00	1.5-2.5	41	16	25	70	CL
40215	12+00	2.5-3.5	41	19	22	75	CL

Table No. 2  
EXPANSIVE PRESSURES AND PERCENT SWELL

Sample No.	Location	Depth ft.	Density #/ft <sup>3</sup>	% - Water	Expansive Pressure #/ft <sup>2</sup>	% - Swell
40133	7+00	0-1.5	106.5	9.9		17.35
"	"	"	"	10.1	4914	
"	"	"	106.4	15.0		12.50
"	"	"	106.0	15.6	4664	
40134	7+00	1.5-2.5	106.4	10.1		9.72
"	"	"	106.4	10.1	2912	
"	"	"	106.4	15.0		5.81
"	"	"	106.0	15.8	1866	
40135	7+00	2.5-3.5	106.5	9.3		5.06
"	"	"	106.5	10.2	1001	
"	"	"	106.4	15.3		4.96
"	"	"	106.3	15.5	819	
40149	8+00	0-1.5	107.1	9.2		9.00
"	"	"	106.6	10.1	2412	
"	"	"	106.6	15.1		6.60
"	"	"	106.6	15.1	2457	
40150	8+00	1.5-2.5	106.3	9.9		11.99
"	"	"	106.8	9.6	2710	
"	"	"	106.9	14.6		8.70
40151	8+00	2.5-3.5	106.4	9.3		9.06
"	"	"	106.9	9.9	1820	
"	"	"	106.5	15.2		4.3
40165	9+00	0-1.0	106.5	9.9		4.78
"	"	"	106.4	10.5	523	
"	"	"	106.4	15.4		2.50
40166	"	1.0-2.0	106.4	9.9		7.37
"	"	"	106.8	9.9	1183	
"	"	"	106.6	14.9		4.90
"	"	"	106.6	14.9	837	
40167	"	2.0-3.0	105.8	9.9		6.99
"	"	"	106.8	10.2	1456	
"	"	"	108.1	13.3		4.40
"	"	"	106.7	15.1	319	
40181	10+00	0-1.0	106.0	9.9		9.66
"	"	"	106.4	9.2	2730	
"	"	"	106.5	15.4	1866	
40182	"	1.0-2.0	106.2	9.2		15.51
"	"	"	106.4	10.5	4368	
"	"	"	106.8	14.6	2730	
40183	"	2.0-3.0	106.1	9.9		10.59
"	"	"	106.2	10.4	3117	
"	"	"	106.4	14.8		10.00
"	"	"	106.4	15.2	2480	
40197	11+00	0-1.5	106.4	10.2		9.49
"	"	"	106.8	9.8	2093	
"	"	"	106.4	15.0		7.60
"	"	"	106.6	15.2	1570	
40198	"	1.5-2.5	106.4	10.2		9.17
"	"	"	106.8	10.0	2275	
"	"	"	106.7	14.9		7.50
"	"	"	106.6	15.2	1456	
40199	11+00	2.5-3.5	107.3	10.5		17.77
"	"	"	106.1	10.4	2975	
"	"	"	106.7	14.8		15.70
"	"	"	106.2	15.8	3822	
40213	12+00	0-1.5	106.7	10.2		6.69
"	"	"	107.4	9.3	1722	
"	"	"	107.0	14.8		3.90
"	"	"	106.7	15.2	728	
40214	"	1.5-2.5	106.7	10.2		11.62
"	"	"	106.5	10.2	2002	
"	"	"	106.0	15.5		8.80
"	"	"	106.4	15.1	1138	
40215	"	2.5-3.5	106.5	10.1		11.05
"	"	"	106.9	10.0	3640	
"	"	"	106.4	15.1		9.5
"	"	"	106.6	15.0	1547	

### Determination of Optimum gmKCl/gm Clay

The next important determination to make was the amount of KCl per unit wt. of clay necessary for proper stabilization of this particular clayey mass. Because of the large mass of data from previous soil stabilization work using this technique it was relatively straightforward to closely estimate the amount of KCl required to reduce the expansive pressure by some reasonable factor.

It was decided to use an expansive pressure reduction factor of 2 as a guideline in determining the overall amount of KCl needed. To use higher factors, say 4 or 5, would result, when coupled with the electrochemical effects induced by the electrical current in the soil, in a very expensive overkill.

Thus, based on previous soil stabilization work a decision as to the gm. wt. of KCl per gm. wt. of clay was made during the logistical build up period.

It was determined from figure 16 that a best estimate from both an engineering and economic standpoint was about 0.02 gm KCl/gm clay. This amount of KCl added to the clay in addition to the fabric changes in the clay, induced by the electrochemical process, was adjudged to be sufficient for our purposes.

# EXPANSIVE PRESSURE VS. % KCL

—#40 SOIL

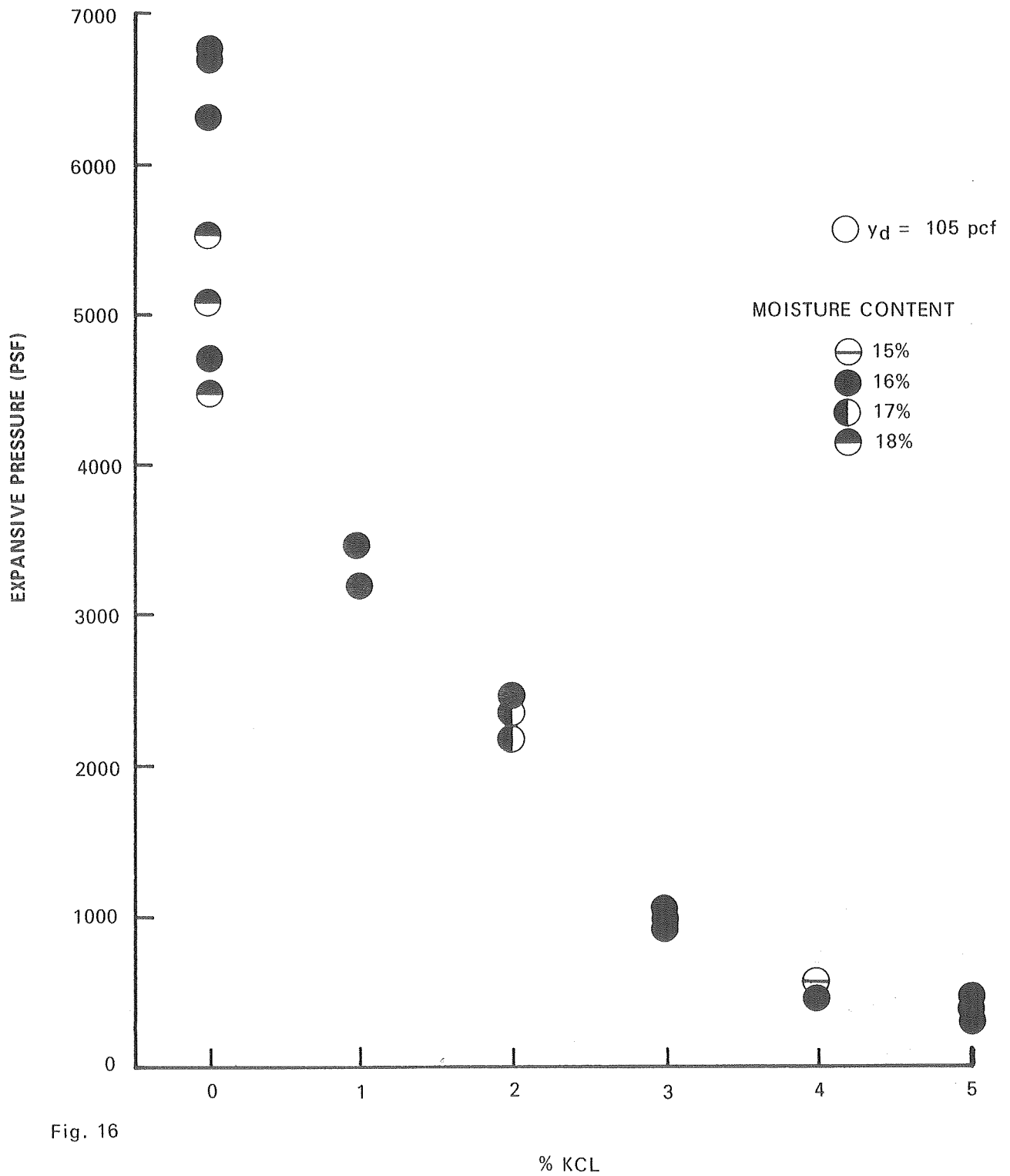


Fig. 16